COLUMN DESIGN FOR FLEXURAL (NON DUCTILE UN-BRACED)

DESIGN CODE ACI 318M-2014

General Data

$ColumnNo$: C9

$Level$: 0 m to 6.2m

$FrameType := 1$ ---------- 1- for Non Ductile, 2 for Special

$B := 600\ mm$ ---------- Width of the Column

$D := 900\ mm$ ---------- Depth of the Column

$f'c := 25\ MPa$ ---------- Grade of Concrete (Cylindrical Strength)

$fy := 420\ MPa$ ---------- Grade of Main Reinforcement

$fyt := 420\ MPa$ ---------- Grade of Secondary Reinforcement

$Cc := 50\ mm$ ---------- Nominal Cover to Beam Tension Reinforcement

$Es := 200000\ MPa$ ---------- Modulus of elasticity of reinforcement

$\phi BM := 0.65$ ---------- Capacity reduction factor for Bending Moment

$\phi shear := 0.75$ ---------- Capacity reduction factor for Shear and torsion

$ptmax := 4$ ---------- Maximum % reinforcement

$ptmin := 1$ ---------- Minimum % reinforcement

$\varepsilon cu := 0.003$ ---------- Permissible Strain in Concrete

$lux := 5400\ mm$ ---------- Clear Floor Height along Major Direction@lux

$luy := 5400\ mm$ ---------- Clear Floor Height along Minor Direction@luy

$NoOfFloors := 1$ ---------- Number of Floors considered after merged of levels

$NoOfColumnsInGroup := 1$ ---------- Number of column in Group

$ColumnType := 1$ ---------- 1- for Braced, 2 for Un-Braced

$MinimumEccentricityCheck$ ---------- Simultaneously (Both Axis)

$CodeDefinedDBRatio := 4$

$$DBRatio := \left\| \begin{array}{l} \text{if } \frac{D}{B} < 4 \\ \quad \| \text{“DesignAsColumn”} \\ \text{else} \\ \quad \| \text{“DesignAsWall”} \end{array} \right\| = \text{“DesignAsColumn”}$$

Reinforcement Provided

$\phi 1 := 25.4\ mm$

$N1 := 12$

$\phi 2 := 19.1\ mm$

$N2 := 8$

$$Ast := \frac{\pi \cdot \phi 1^2}{4} \cdot N1 + \frac{\pi \cdot \phi 2^2}{4} \cdot N2 = 8372.659\ mm^2$$ ---------- Area of Reinforcement Provided

Flexural Design (Analysis Forces)

$AnalysisReferenceNo$ 4040

$LoadCombination$ [8] : 5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD CASE 2) - (LOAD 3: LOAD CASE 3 EQ-X)

$CriticalLoacation$ Top Joint

$Put := 2751.17\ kN$ ---------- Ultimate moment from analysis at Top Node

$Muxt := 108.8\ kN \cdot m$ ---------- Ultimate moment from analysis along D at Top Node
$Muyt := -973.49\ kN \cdot m$ ---------- Ultimate moment from analysis along B at Top Node
$Vuxt := -227.03\ kN$ ---------- Ultimate Shear from analysis along D at Top Node
$Vuyt := -51.96\ kN$ ---------- Ultimate Shear from analysis along B at Top Node
$Pub := 2751.17\ kN$ ---------- Ultimate moment from analysis at Bottom Node
$Muxb := -213.27\ kN \cdot m$ ---------- Ultimate moment from analysis along D at Bottom Node
$Muyb := 433.74\ kN \cdot m$ ---------- Ultimate moment from analysis along B at Bottom Node
$Vuxb := -227.03\ kN$ ---------- Ultimate Shear from analysis along D at Bottom Node
$Vuyb := -51.96\ kN$ ---------- Ultimate Shear from analysis along B at Bottom Node

Sway Calculation (Stability Index)

For_Global_X_Direction

LoadCombination [13] : 1.2(LOAD 1: LOAD CASE 1) +1.2 (LOAD 2: LOAD CASE 2)-1.2 (LOAD 3: LOAD CASE 3 EQ-X)

$StoreyHeight(A) := 6200\ mm$ ---------- Height of Story Considered for design
$GravityLoad(B) := 61274.48\ kN$ ---------- Summation of Axial Load of Story selected
$RelativeDisplacement(C) := 3.21\ mm$ ---------- Relative Displacement for Story selected
$StoreyShear(D) := 400.16\ kN$ ---------- Story Shear for Story selected

$$StabilityIndex_QD := \frac{GravityLoad(B)}{StoreyHeight(A)} \cdot \frac{RelativeDisplacement(C)}{StoreyShear(D)} = 0.0793$$

---------- Clause 6.6.4.4.1

$$Check := \left\| \begin{array}{l} \text{if } StabilityIndex_QD \le 0.05 \\ \quad \| \text{“Non Sway Frame(Braced)”} \\ \text{else} \\ \quad \| \text{“Sway Frame(UnBraced)”} \end{array} \right. = \text{“Sway Frame(UnBraced)”}$$

---------- Clause 6.6.4.3

For_Global_Y_Direction

LoadCombination [13] : 1.2(LOAD 1: LOAD CASE 1) +1.2 (LOAD 2: LOAD CASE 2)+1.2 (LOAD 3: LOAD CASE 3 EQ-Y)

$StoreyHeight(A) := 6200\ mm$ ---------- Height of Story Considered for design
$GravityLoad(B) := 61274.48\ kN$ ---------- Summation of Axial Load of Story selected
$RelativeDisplacement(C) := 2.3\ mm$ ---------- Relative Displacement for Story selected
$StoreyShear(D) := 300.12\ kN$ ---------- Story Shear for Story selected

$$StabilityIndex_QB := \frac{GravityLoad(B)}{StoreyHeight(A)} \cdot \frac{RelativeDisplacement(C)}{StoreyShear(D)} = 0.076$$

$$Check := \left\| \begin{array}{l} \text{if } StabilityIndex_QB \le 0.05 \\ \quad \| \text{“Non Sway Frame(Braced)”} \\ \text{else} \\ \quad \| \text{“Sway Frame(UnBraced)”} \end{array} \right. = \text{“Sway Frame(UnBraced)”}$$

---------- Clause 6.6.4.3

Effective Length Factor

$EffectiveLengthFactorAlongD := 1.66$
$EffectiveLengthFactorAlongB := 1.32$

Calculation of Slenderness Check

Along D:

$EffectiveLengthD := EffectiveLengthFactorAlongD \cdot luy = 8964\ mm$

$A := D \cdot B = 540000\ mm^2$ ---------- Area of Cross Section

$IxxD := B \cdot \frac{D^3}{12} = (3.645 \cdot 10^{10})\ mm^4$ ---------- Moment of Inertia

$rD := \sqrt{\left(\frac{IxxD}{A}\right)} = 259.808\ mm$ ---------- Radius of Gyration

$k_D := EffectiveLengthFactorAlongD = 1.66$

$Actual_Slender_D := \frac{EffectiveLengthD}{rD} = 34.502$

$LerPerD := 22$ ---------- Clause 6.2.5.1(a)

```
Check1 := | if LerPerD > Actual_Slender_D                    | = "Slender_Use Approximate method"
          |   | "Column Not Slender Along B"                |
          | else if Actual_Slender_D < 100                  |
          |   | "Slender_Use Approximate method"            |
          | else                                            |
          |   | "Slender_P-delta effect (revise)"           |
```

Along B:

$EffectiveLengthB := EffectiveLengthFactorAlongB \cdot lux = 7128\ mm$

$A := D \cdot B = 540000\ mm^2$ ---------- Area of Cross Section

$IxxB := D \cdot \frac{B^3}{12} = (1.62 \cdot 10^{10})\ mm^4$ ---------- Moment of Inertia

$rB := \sqrt{\left(\frac{IxxB}{A}\right)} = 173.205\ mm$ ---------- Radius of Gyration

$Actual_Slender_B := \frac{EffectiveLengthB}{rB} = 41.154$

$LerPerB := 22$ ---------- Clause 6.2.5.1(a)

```
Check1 := | if LerPerB > Actual_Slender_B                    | = "Slender_Use Approximate method"
          |   | "Column Not Slender Along B"                |
          | else if Actual_Slender_B < 100                  |
          |   | "Slender_Use Approximate method"            |
          | else                                            |
          |   | "Slender_P-delta effect (revise)"           |
```

Calculation of Slenderness Moment - Un-Braced (Sway) Frame

$\beta dns := 0.6$ ---------- Clause 6.6.4.4.4

$Ec := 4700 \cdot \sqrt{\left(\frac{f'c}{MPa}\right)} = 23500$ ---------- Clause 19.2.2.1

$Ig := D \cdot \frac{B^3}{12} = \left(1.62 \cdot 10^{10}\right)\ mm^4$ ---------- Moment of Inertia of Section

$EI := 0.288675 \cdot Ec \cdot Ig = \left(1.099 \cdot 10^{14}\right)\ mm^4$

$$\delta ns := \left\| \begin{array}{l} \text{if } StabilityIndex_QD > 1 \\ \quad \| \ 1.5 \\ \text{else} \\ \quad \| \ \frac{1}{\left(1 - StabilityIndex_QD\right)} \end{array} \right\| = 1.086$$

---------- Clause 6.6.4.5.3

Along D:

$M1s := 20.42\ kN \cdot m$ ---------- factored end moment on compression member at the end at which M1 acts, due to loads that cause appreciable side-sway. Clause 6.6.4.6

$M2s := -211.38\ kN \cdot m$

Columns	Level	Analysis No	Load Case	Location	P	Mx	My	ShearX	ShearY	Torsion
				(m)	(kN)	(kNm)	(kNm)	(kN)	(kN)	(kNm)
C9	6.2 m	4040	LOAD 1: LOAD CASE 1	0	-11.54	0.22	-0.09	2.53	-1.1	0
			LOAD 1: LOAD CASE 1	6.2	-11.54	7.04	15.57	2.53	-1.1	0
			LOAD 2: LOAD CASE 2	0	561.83	-0.6	87.36	-48.06	-1.81	0
			LOAD 2: LOAD CASE 2	6.2	561.83	10.64	-210.55	-48.06	-1.81	0
			LOAD 3: LOAD CASE 3 EQ-X	0	0.27	211.38	2.59	-0.65	37.4	0
			LOAD 3: LOAD CASE 3 EQ-X	6.2	0.27	-20.42	-1.42	-0.65	37.4	0
			LOAD 4: LOAD CASE 4 EQ-Y	0	11.77	-0.11	91.01	-23.8	-0.02	0
			LOAD 4: LOAD CASE 4 EQ-Y	6.2	11.77	0.01	-56.53	-23.8	-0.02	0

The above value is from the Basic load case

$\delta sM1s := \delta ns \cdot M1s = 22.178\ kN \cdot m$

$\delta sM2s := \delta ns \cdot M2s = -229.581\ kN \cdot m$

$M1 := 108.8\ kN \cdot m$

$M2 := -213.27\ kN \cdot m$

$M1ns := \left(M1 - M1s\right) = 88.38\ kN \cdot m$

$M2ns := \left(M2 - M2s\right) = -1.89\ kN \cdot m$

factored end moment on compression member at the end at which M1 acts, due to loads that cause no appreciable sides-way. Clause 6.6.4.6

Columns	Level	Analysis No	Load Comb	Load Comb	Load Comb	Location	P	Mx	My	ShearX	ShearY	Torsion
			Ref No	Analysis No	Description	(m)	(kN)	(kNm)	(kNm)	(kN)	(kN)	(kNm)
C9	6.2 m	4040	1	100	1.4 (LOAD 1: LOAD CASE 1)	0	-16.15	0.31	-0.13	3.54	-1.54	0
			1	100	1.4 (LOAD 1: LOAD CASE 1)	6.2	-16.15	9.85	21.8	3.54	-1.54	0
			2	101	1.2 (LOAD 1: LOAD CASE 1) +1.6 (LOAD 2: L(	0	885.08	-0.69	139.66	-73.87	-4.22	0
			2	101	1.2 (LOAD 1: LOAD CASE 1) +1.6 (LOAD 2: L(	6.2	885.08	25.47	-318.2	-73.87	-4.22	0
			3	102	1.2 (LOAD 1: LOAD CASE 1) +0.5 (LOAD 2: L(	0	267.07	-0.04	43.57	-21	-2.23	0
			3	102	1.2 (LOAD 1: LOAD CASE 1) +0.5 (LOAD 2: L(	6.2	267.07	13.76	-86.59	-21	-2.23	0
			4	103	1.2 (LOAD 1: LOAD CASE 1)	0	-13.85	0.26	-0.11	3.03	-1.32	0
			4	103	1.2 (LOAD 1: LOAD CASE 1)	6.2	-13.85	8.44	18.69	3.03	-1.32	0
			6	105	4 (LOAD 1: LOAD CASE 1) +4 (LOAD 2: LOAD	0	2201.42	209.87	351.66	-182.79	25.75	0
			6	105	4 (LOAD 1: LOAD CASE 1) +4 (LOAD 2: LOAD	6.2	2201.42	50.28	-781.35	-182.79	25.75	0
			7	106	4 (LOAD 1: LOAD CASE 1) +4 (LOAD 2: LOAD	0	2212.92	-1.62	440.07	-205.95	-11.67	0
			7	106	4 (LOAD 1: LOAD CASE 1) +4 (LOAD 2: LOAD	6.2	2212.92	70.71	-836.46	-205.95	-11.67	0
			8	107	5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD	0	2751.17	-213.27	433.74	-227.03	-51.96	0
			8	107	5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD	6.2	2751.17	108.8	-973.49	-227.03	-51.96	0
			9	108	5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD	0	2739.67	-1.78	345.33	-203.88	-14.54	0
			9	108	5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD	6.2	2739.67	88.37	-918.38	-203.88	-14.54	0

$M1sldr := M1ns + \delta sM1s = 110.558\ kN \cdot m$

$1.4 \cdot Muxt = 152.32\ kN \cdot m$ ---------- Clause 6.2.6

$M1sldr_D := min(\text{abs}(1.4 \cdot Muxt), \text{abs}(M1sldr)) \cdot \text{sign}(M1sldr) = 110.558\ kN \cdot m$

$M2sldr := M2ns + \delta sM2s = -231.471\ kN \cdot m$

$1.4\ Muxb = -298.578\ kN \cdot m$

$M2sldr_D := min(\text{abs}(1.4 \cdot Muxb), \text{abs}(M2sldr)) \cdot \text{sign}(M2sldr) = -231.471\ kN \cdot m$

Along B:

$M1s := 1.42\ kN \cdot m$ ---------- factored end moment on compression member at the end at which M1 acts, due to loads that cause appreciable side-sway. Clause 6.6.4.6

$M2s := -2.59\ kN \cdot m$

$\delta sM1s := \delta ns \cdot M1s = 1.542\ kN \cdot m$

$\delta sM2s := \delta ns \cdot M2s = -2.813\ kN \cdot m$

$M1 := -973.49\ kN \cdot m$

$M2 := 433.74\ kN \cdot m$

$M1ns := (M1 - M1s) = -974.91\ kN \cdot m$ factored end moment on compression member at the end at which M1 acts, due to loads that cause no appreciable sides-way. Clause 6.6.4.6

$M2ns := (M2 - M2s) = 436.33\ kN \cdot m$

$M1sldr := M1ns + \delta sM1s = -973.368\ kN \cdot m$ ---------- Clause 6.2.6

$1.4 \cdot Muyt = -1362.886\ kN \cdot m$ ---------- Clause 6.2.6

$M1sldr_B := min(\text{abs}(1.4 \cdot Muyt), \text{abs}(M1sldr)) \cdot \text{sign}(M1sldr) = -973.368\ kN \cdot m$

$M2sldr := M2ns + \delta sM2s = 433.517\ kN \cdot m$

$1.4\ Muyb = 607.236\ kN \cdot m$

$M2sldr_B := min(\text{abs}(1.4 \cdot Muyb), \text{abs}(M2sldr)) \cdot \text{sign}(M2sldr) = 433.517\ kN \cdot m$

Final Design Forces for Flexural Design

At Top Node:

$Put = 2751.17\ kN$

$Mux := M1sldr_D = 110.558\ kN \cdot m$

$Muy := M1sldr_B = -973.368\ kN \cdot m$

At Bottom Node:

$Pub = 2751.17\ kN$

$Mux := M2sldr_D = -231.471\ kN \cdot m$

$Muy := M2sldr_B = 433.517\ kN \cdot m$

Final Critical Design Forces

At Top Node:

$Put = 2751.17\ kN$

$Mux_Final := M1sldr_D = 110.558\ kN \cdot m$

$Muy_Final := M1sldr_B = -973.368\ kN \cdot m$

ϕPn, Max Check

$fy_1 := fy = 420\ MPa$ ---------- Clause 22.4.2.1

$LoadCombination$ [8] : 5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD CASE 2) - (LOAD 3: LOAD CASE 3 EQ-X))

$CriticalLocation$ Bottom Joint

$Pu := 2751.17\ kN$ ---------- Factored Axial force

$Mux1 := -213.24\ kN \cdot m$ ---------- Factored Bending Moment along Major Direction

$Muy1 := 433.74\ kN \cdot m$ ---------- Factored Bending Moment along Minor Direction

$Ag := B \cdot D = 540000\ mm^2$ ---------- Cross Section Area

$Ast = 8372.659\ mm^2$ ---------- Area of Reinforcement Provided

$pt := \frac{Ast \cdot 100}{Ag} = 1.55$ ---------- % Reinforcement Provided

$\phi Pnmax := 0.8 \cdot \phi BM \cdot (0.85 \cdot f'c \cdot (Ag - Ast) + (Ast \cdot fy_1)) = 7703.071\ kN$

---------- Clause 22.4.2.2

$$Check := \left\| \begin{array}{l} \text{if } Pu \le \phi Pnmax \\ \quad \| \text{"Ok"} \\ \text{else} \\ \quad \| \text{"Revise"} \end{array} \right\| = \text{"Ok"}$$

Minimum Ast Calculation

$fy_2 := fy = 420\ MPa$ ---------- Clause 22.4.2.1

$LoadCombination$ [8] : 5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD CASE 2) - (LOAD 3: LOAD CASE 3 EQ-X)

$Pu_max := 2751.17\ kN$

$$AstReq := \frac{0.442 \cdot f'c \cdot Ag - Pu_max}{(0.442 \cdot f'c) - (0.52 \cdot fy_2)} = -1.551 \cdot 10^4\ mm^2$$

---------- Clause 22.4.2.2

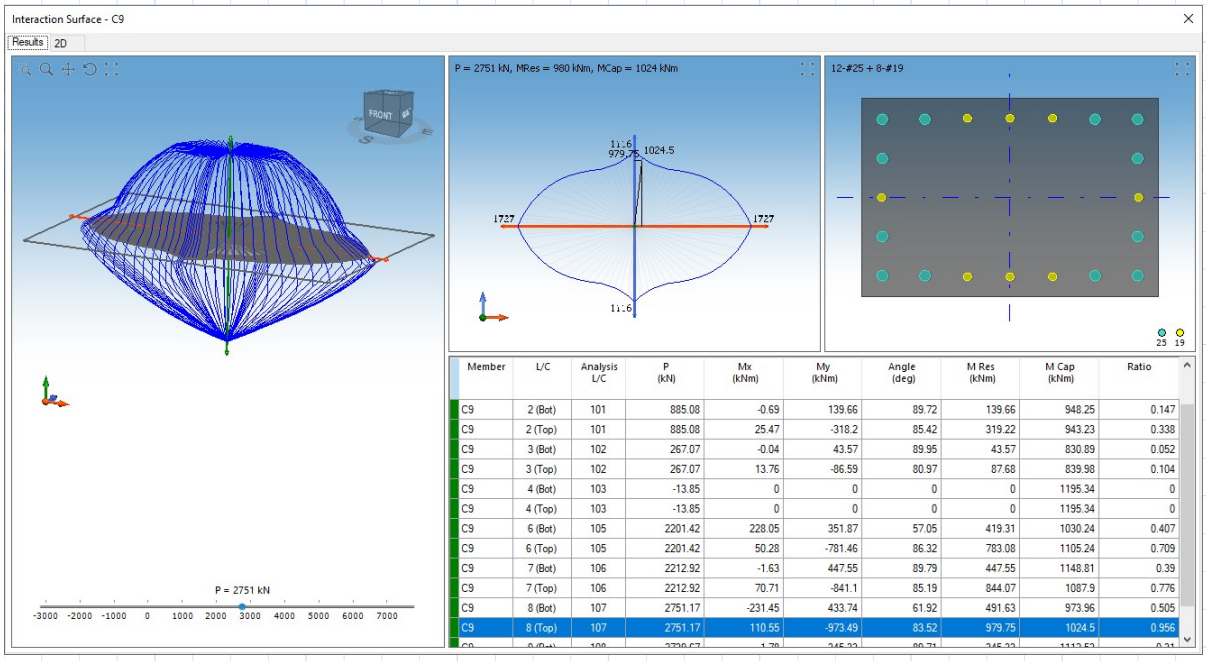
$$pt := \left\| \begin{array}{l} \text{if } AstReq \le 0 \\ \quad \| 0 \\ \text{else} \\ \quad \| \frac{AstReq}{Ag} \end{array} \right\| = 0$$

$ptmin := \max(0.5, pt) = 0.5$ ---------- Clause 10.3.1.2

$UserDefinedpt := 1$

$Finalpt := \max(ptmin, UserDefinedpt) = 1$

PM Curve



Member	L/C	Analysis L/C	P (kN)	Mx (kNm)	My (kNm)	Angle (deg)	M Res (kNm)	M Cap (kNm)	Ratio
C9	2 (Bot)	101	885.08	-0.69	139.66	89.72	139.66	948.25	0.147
C9	2 (Top)	101	885.08	25.47	-318.2	85.42	319.22	943.23	0.338
C9	3 (Bot)	102	267.07	-0.04	43.57	89.95	43.57	830.89	0.052
C9	3 (Top)	102	267.07	13.76	-86.59	80.97	87.68	839.98	0.104
C9	4 (Bot)	103	-13.85	0	0	0	0	1195.34	0
C9	4 (Top)	103	-13.85	0	0	0	0	1195.34	0
C9	6 (Bot)	105	2201.42	228.05	351.87	57.05	419.31	1030.24	0.407
C9	6 (Top)	105	2201.42	50.28	-781.46	86.32	783.08	1105.24	0.709
C9	7 (Bot)	106	2212.92	-1.63	447.55	89.79	447.55	1148.81	0.39
C9	7 (Top)	106	2212.92	70.71	-841.1	85.19	844.07	1087.9	0.776
C9	8 (Bot)	107	2751.17	-231.45	433.74	61.92	491.63	973.96	0.505
C9	8 (Top)	107	2751.17	110.55	-973.49	83.52	979.75	1024.5	0.956

Resultant Moment (Combined Action)
Moment Capacity Check

$$Ptcalculated := \frac{Ast \cdot 100}{Ag} = 1.55$$ ---------- % Reinforcement Provided

$$LoadAngle := 83.52$$ ---------- tan-1 (Muy_Final / Mux_Final)

$$Mres := \sqrt{Mux_Final^2 + Muy_Final^2} = 979.626\ kN \cdot m$$

---------- Resultant Bending Moment

$$\phi Mcap := 1024.5\ kN \cdot m$$ ---------- Moment Capacity from PM Curve

$$CapacityRatio := \frac{Mres}{\phi Mcap} = 0.956$$ ---------- Clause 10.4.2

$$Check := \left\| \begin{array}{l} \text{if } CapacityRatio < 1 \\ \quad \| \text{"Ok"} \\ \text{else} \\ \quad \| \text{"Revise"} \end{array} \right\| = \text{"Ok"}$$ ---------- Clause 10.4.2

Design Data for All Load Combination

Column/Wall	:	C9	
Level	:	0 m To 6.2 m	
Frame Type	=	Non-Ductile	
Design Code	=	ACI 318M - 14	
Grade Of Concrete (f'c)	=	25	N/sqmm
Grade Of Steel (Main)	=	420	N/sqmm
Grade Of Steel (Shear)	=	420	N/sqmm
Column B	=	600	mm
Column D	=	900	mm
Clear Cover	=	50	mm
Pt	=	1.55	%

Design Table :

Member	LOC	L/C	Analysis L/C No	Pu	Analysis		Msldr or MC		Design			Mcap	Capacity Ratio
					Mx	My	Mx	My	Mux	Muy	MuRes		
				(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(kNm)	(kNm)	(kNm)	(kNm)	
4040	BOT	1	100	-16.15	0.31	-0.13	-	-	0	0	0	1194.61	0
4040	TOP	1	100	-16.15	9.85	21.8	-	-	0	0	0	1194.61	0
4040	BOT	2	101	885.08	-0.69	139.66	-0.69	139.66	-0.69	139.66	139.66	948.25	0.147
4040	TOP	2	101	885.08	25.47	-318.2	25.47	-318.2	25.47	-318.2	319.22	943.23	0.338
4040	BOT	3	102	267.07	-0.04	43.57	-0.04	43.57	-0.04	43.57	43.57	830.89	0.052
4040	TOP	3	102	267.07	13.76	-86.59	13.76	-86.59	13.76	-86.59	87.68	839.98	0.104
4040	BOT	4	103	-13.85	0.26	-0.11	-	-	0	0	0	1195.34	0
4040	TOP	4	103	-13.85	8.44	18.69	-	-	0	0	0	1195.34	0
4040	BOT	6	105	2201.42	209.87	351.66	228.05	351.87	228.05	351.87	419.31	1030.24	0.407
4040	TOP	6	105	2201.42	50.28	-781.35	50.28	-781.46	50.28	-781.46	783.08	1105.24	0.709
4040	BOT	7	106	2212.92	-1.62	440.07	-1.63	447.55	-1.63	447.55	447.55	1148.81	0.39
4040	TOP	7	106	2212.92	70.71	-836.46	70.71	-841.1	70.71	-841.1	844.07	1087.9	0.776
4040	BOT	8	107	2751.17	-213.27	433.74	-231.45	433.74	-231.45	433.74	491.63	973.96	0.505
4040	TOP	8	107	2751.17	108.8	-973.49	110.55	-973.49	110.55	-973.49	979.75	1024.5	0.956
4040	BOT	9	108	2739.67	-1.78	345.33	-1.78	345.33	-1.78	345.33	345.33	1112.53	0.31
4040	TOP	9	108	2739.67	88.37	-918.38	88.37	-918.38	88.37	-918.38	922.62	1035.43	0.891